CHARACTERIZATION OF DESIGN PROPERTIES (COMPRESSIVE STRENGTH AND RESILIENT MODULUS) OF LIME, CEMENT, FLY ASH STABILIZED STRUCTURAL RECYCLED CONCRETE BASE AS A FUNCTION OF CURING TIME

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Abstract

Taxiway WP is a new parallel taxiway at Bush Intercontinental Airport in Houston, Texas, with an innovative design philosophy. The pavement structure consists of a cement and fly ash stabilized subgrade overlain by 18-inches of lime-cement-fly ash stabilized recycled crushed concrete (LCFRCC) and surfaced with 14-inches of PCC. An asphalt bond breaker is used between the PCC and the LCFRCC. The LCFRCC was designed to provide a resilient modulus of 1,000,000 psi when mature (after approximately one year of field curing). The LCFRCC was designed to develop a long, slow strength which will minimize volume changes due to curing (minimize differential movement between the PCC and the LCFRCC) and maximize and extend damage recovery as a result of autogeneous healing. Long-term laboratory testing demonstrated the slow controlled pozzolanic strength gain provided by the LCFRCC. A stabilizer combination of 10 percent class C fly ash, 3 percent hydrated lime, and 0.5 percent Portland cement was selected from several combinations to provide the desired ultimate level of strength gain and rate of strength gain. The accompanying research demonstrates the success of the mixture in achieving the desired goals. A correlation between compressive strength gain and increase in resilient modulus as a function of time of curing was developed. The correlation between compressive strength and modulus is compared to similar correlations in the literature and represents the importance of developing a unique relationship between these two parameters and the influence of aggregate properties and stabilizer matrix properties.

Introduction

The Houston Airport System is currently undertaking a $ 2.6 billion capital improvement program (CIP) to expand the capacity of the Houston tri-airport system (Bush Intercontinental Airport-BIAH, Hobby Airport, and Ellington Field) to accommodate forecasted increases in air traffic. The CIP includes upgrades to existing facilities and new facilities both on the landside and airside at all three airports. One of the major improvements to expand air traffic capacity is the expansion and widening of Runway 15R-33L, which is one of four runways currently in operation at BIAH. Built in 1982, Runway 15R-33L is a Group III VFR runway constructed of asphalt concrete pavement. The runway, being VFR, 100-feet wide by 6,038-feet long, cannot support larger aircraft and its utilization at BIAH is constrained to smaller General Aviation aircraft. Therefore, as a part of the overall improvement program at BIAH Runway 15R-33L is being upgraded to a Category I precision runway, 150-feet wide by 10,000-feet long, capable of handling Group V carrier aircraft. In support of the upgraded runway approximately 6 miles of new taxiways are being built including a new parallel Taxiway WP; taxiway extensions; high speed exits; aircraft hold areas; and necessary drainage, utility, and service road improvements (see figure 1 for layout of improvements). The total construction cost of the Runway 15R-33L extension, widening, and associated taxiways is approximately $81 million.

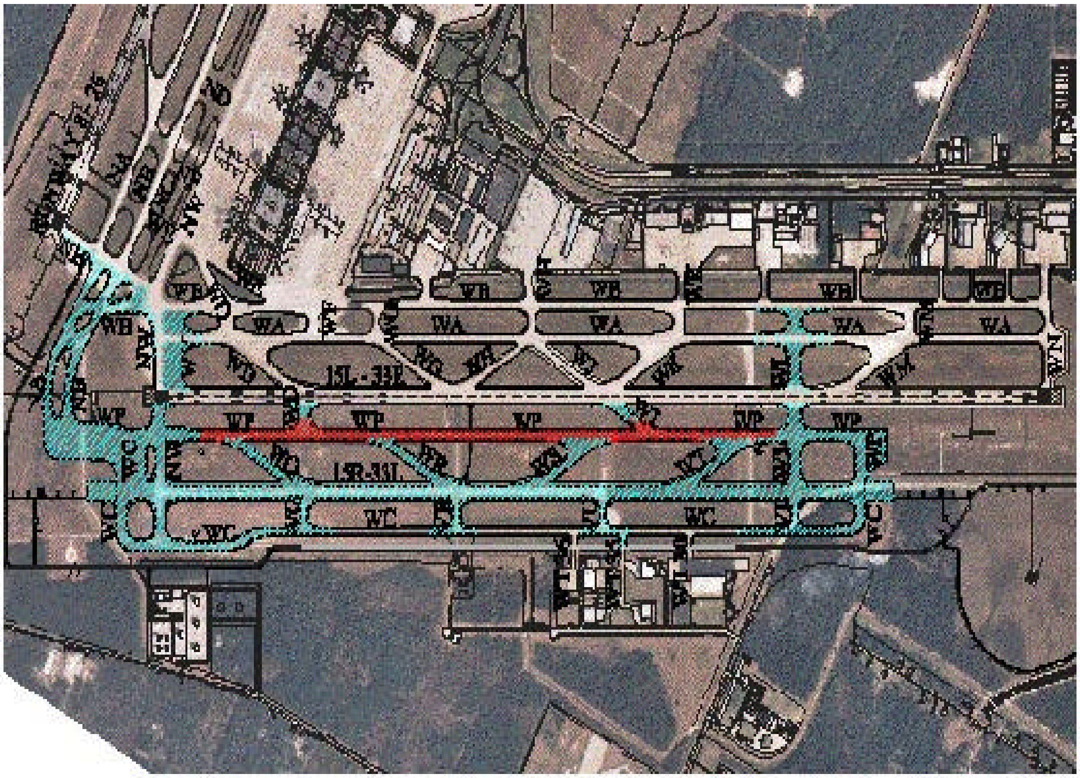

Figure 1. General Layout of Improvements at BIAH.

This paper describes the application of a lime-cement-fly ash (LCF) subbase used in taxiway WP, the parallel taxiway for runway 15R-33L. LCF technology was developed by Nai Yang and was originally used at Newark International Airport in 1969, then at Portland International Airport in 1974, and subsequently at Zurich, Switzerland, International Airport in 1979. In 1986 over 1,000,000 tons of LCF were used in building the south complex at BIAH. This complex consists of runway 9-27 and supporting taxiways SA and SB. Runway 9-27 was rehabilitated in 1998. The engineering evaluation that accompanied rehabilitation consisted of an extensive analysis of the LCF, and the evaluation proved that the LCF was in excellent condition and had functioned as designed over the 10-year period.

This paper describes the role of LCF in taxiway WP. The paper specifically discusses the maximization of the use of recycled materials in the mixture design, target strengths and moduli of the design LCF mixture, and the expected performance of the LCF layer in taxiway WP.

Use of LCF at BIAH: Background

Runway 9-27, as originally constructed, consisted of 28-inch of LCF with a 3-inch hot mix asphalt (HMA) surface. The LCF rested on a 18-inches of embankment material and 8-inches of cement stabilized subgrade. The LCF was designed to provide a long-term strength gain, which was validated through laboratory and field-testing. Laboratory testing of samples subjected to accelerated cure, designed to duplicate approximately 6-months of field cure, gave strengths of approximately 1,200 psi. The average strength of 30 LCF cores collected in 1996, approximately 10 years after construction, yielded an average unconfined compressive strength of approximately 3,014 psi with a coefficient of variation of approximately 19 percent. Modulus of elasticity determined on five samples averaged approximately 3,500,000 psi. Thus the strength more than doubled between the six-month period and the 10-year period. This is evidence of the desired long-term, continued strength development. Pozzolanic reactions that facilitate these long-term strength gains also promote autogenous healing, which is responsible for healing of areas of microcracks leading to extended fatigue life. The LCF used on runway 9-27 was generally comprised of 73.5 percent sand and gravel, 12.5 percent bank sand, 9.5 percent class C fly ash, 4.0 percent hydrated lime, and 0.5 percent Portland cement.

In 1996 runway 9-27 was scheduled for rehabilitation. The structural quality of the LCF was carefully assessed and found to be excellent. An engineering analysis (Godiwalla et al, 2000) determined that a 5-inch polymer modified asphalt overlay was sufficient to restore ride quality and provide a 20-year service life. Prior to placement of the 5-inch overlay, the existing 3-inch HMA surface was milled to a depth of approximately 1-inch to assure a strong bond with the overlay. Then a stress absorbing membrane interlayer was placed to minimize the probability of propagation of reflective cracks from the existing surface. The thickness of the overlay was predicated based on the horizontal shearing stresses transmitted (by braking and turning aircraft at taxiway exits) to the stress absorbing membrane interlayer. High shear stresses can exacerbate the potential of the overlay to deform or rut under such traffic.

The engineering analysis prior to rehabilitation in 1996 demonstrated that the LCF had continually gained strength over the performance period, which substantiates that pozzolanic strength gain continued during this period supporting autogenous healing. Reflective cracking from the LCF into the HMA surface was present but not extensive. Prior to placement of the original HMA surface, joints were saw-cut into the LCF and through the HMA every 200 feet to accommodate shrinkage cracking. Originally expansion joints were constructed in 1996. The 1998 overlay is functioning very well.

Adapting LCF to Taxiway WP

Several options were considered for the design of taxiway WP including the use of an LCF subbase. This design is only attractive if it results in cost savings. Designers decided to use readily available recycled crushed concrete (RCC) as the aggregate in the LCF mixture. Large volumes of RCC had been stockpiled less than ten miles from the runway project by Southern Crushed Concrete. The RCC is widely used by both the Houston Airport System and the Texas Department of Transportation, TxDOT.

Designers planned to use the LCF as a subbase below the Portland Cement Concrete (PCC) surface. A stress absorbing membrane interlay (SAMI) was included in the design to reduce the potential for bonding between the PCC surface and the LCF subbase. The PCC and LCF are dissimilar materials in terms of thermal volume change characteristics and hydration shrinkage characteristics; therefore, a bond-breaker was considered a necessity. Design of the PCC/LCF system was based on an assumption that that the surface and base act as separate layers and are not bonded. This diminishes the structural contribution of the LCF to some extent, but is a more appropriate design.

In the design of runway 9-27, the LCF performed as the major structural layer with only a 3-inch HMA surface. The 28-inch LCF functioned well for this purpose. The application of LCF in taxiway WP has a very different application. While it does function as a structural subbase, it is not the major structural layer. That, of course, is the PCC surface. Furthermore, very stiff subbases can actually exacerbate the curling and warping stresses induced in the PCC surface as they form a rigid sublayer below the deformed PCC slab. Therefore, it is appropriate to design the LCF with enough strength and a high enough elastic modulus to provide the necessary structural support and still not be overly rigid so as to exacerbate curling and warping stresses. Based on these considerations, the pavement team designed for a target elastic modulus of 1,000,000 psi for the mature LCF, with an interim (6-month) target modulus of approximately 400,000 psi.

Preliminary Pavement Design

A preliminary design was performed for the design traffic mix using FAA's layered elastic computer program, LEDFAA. The materials selected for the analysis were PCC surface, LCF subbase (with a modulus of 400,000 psi from the initiation of traffic until one year of service and 1,000,000 psi thereafter), and cement fly ash (CFA) stabilized subgrade (with a modulus increasing from 30,000 psi between the initiation of traffic and gradually increasing to 150,000 after 2-years). The natural subgrade is a sandy silt with an average annual design resilient modulus of approximately 10,000 psi. According to the LEDFAA analysis for an unbonded condition, the candidate pavement sections were:

12-inches of PCC; 20-inches of LCF; 8-inches of CFA
14-inchs of PCC; 18-inches of LCF; 8-inches of CFA
15-inches of PCC; 15-inches of LCF; 8-inches of CFA
16-inches of PCC; 13-inches of LCF; 8-inches of CFA
17-inches of PCC; 10-inches of LCF; 8-inches of CFA

The LEDFAA candidate sections were further analyzed using a finite element model (FEM). In this analysis, the critical aircraft, a Boeing 737-800, was used. The FEM was used to calculate interior, corner and edge stresses for 20-feet by 20-feet PCC panels, which were used in lieu of the more typical 25-feet by 25-feet panels to minimize curling and warping stresses over the stiff subbase. As would be expected, edge stresses were critical. Table 1 summarizes the stresses calculated in the FEM analysis for the mature pavements, e.g., when LCF has developed a resilient modulus of 1,000,000 psi at maturity of at one year of service.

Table 1. Summary of Load and Curling Stresses Induced in PCC from FEM Analysis.

PCC Layer Thickness, Inches (E_{PCC} = 4,000,000 psi	**LCF Layer Thickness, Inches (E_{LCF} = 1,000,000 psi)**	**Load Induced Edge Stress, psi**	**Load and Curling Edge Stress, psi**
8	24	301	401
	18	417	517
	12	611	711
10	24	285	385
	18	364	464
	12	416	516
12	24	252	352
	18	322	422
	12	392	492
14	24	241	341
	18	287	387
	12	323	423

A fatigue consumption analysis was performed for both the immature (less than one-year of service) and mature (greater than one-year of service) pavements. Fatigue damage for the immature pavement was negligible; therefore, the results in Table 1 only reflect the stresses induced in the mature pavement (one to 30-year analysis period). Equation (1) (developed by Darter and Barenberg, 1977) was used as the transfer function that relates stress state in the form of the stress ratio or SR (induced stress divided by flexural strength or rupture modulus of the Portland Cement Concrete) to the number of load applications to failure, N.

$$\log N = 16.61 - 17.61\,(SR) \tag{1}$$

A much more conservative transfer function was developed by the PCA (Packard and Tayabji, 1985). However, the PCA approach is to neglect the effect of curling stresses in the design. The stresses used in the analysis were either edge stresses for non-critical periods when curling effects were either zero or not additive or the sum of edge stresses and curling stresses when curling stresses were additive with loads stresses (critical periods). The design team assumed that critical stress levels are developed under about 25 percent of the traffic, and the remainder are non-critical.

The calculated edge stresses were adjusted using a dimensionless analysis technique developed by Zollinger et al. (1989) to account for the effect of load transfer efficiency (LTE). Table 2 shows the effect of LTE on the edge stress reduction using the approach of Zollinger et al. For example, for a load transfer efficiency of 50 percent, the edge stress is 0.85 times the stress calculated assuming no load transfer or a free edge condition. In taxiway WP, all transverse joints are doweled and designed to provide a LTE of at least 85 percent resulting in an edge stress reduction factor of 66 percent. In other words, the calculated edge stress is multiplied by 0.66 to calculate the stress due to the efficient load transfer system.

Table 2. Effect of LTE on Edge Stress Reduction.

LTE, %	**Edge Stress Reduction Factor**
50	85
85	66
90	60

Selection of a design rupture modulus is a key in the design. A common practice is to use 110 percent of the 90-day flexural strength. Based on historical data from flexural beam tests at BIAH and the fact that a 650 psi rupture modulus at 28-days of cure is required by specification, we used a design rupture modulus of 700 psi. Based on this value, the concomitant stress ratio was calculated, as was the predicted fatigue life consumption (calculated from equation 1). The cells shaded in Table 1 represent unacceptable designs. Furthermore, the design team considered a PCC thickness of less than 12-inches to be unacceptable based on engineering judgment. Since the 14-inch PCC; 18-inch LCF; 8-inch CFA design was also acceptable based on LEDFAA analysis, it was selected as the preliminary design. According to the FEM analysis, the design traffic on taxiway WP will consume a less than one percent of the fatigue life of the PCC pavement surface.

The preliminary analysis was approached in an alternative manner as well. We considered the method developed by Packard (1973) where a structural transformation of the base layer is made based on the ratio of the radii of relative stiffness for the PCC and LCF subbase. According to Parkard, the relative change in stiffness of a PCC slab due to the presence of a stabilized subbase, r, taken to the 1.33 power ($r^{1.33}$) and multiplied by the thickness of the PCC slab represents the transformed thickness due to the subbase. In the case of the 14-inch PCC; 18-inch LCF section, where the design elastic modulus of the LCF is 1,000,000 psi, the equivalent PCC slab thickness is approximately 19.25-inches. If the ultimate design modulus of the LCF achieved only the 6-month level of stiffness (500,000 psi), the transformed thickness would be 18-inches of PCC. This is approximately equivalent to the popularly used design of 17-inches of PCC over 6-inches of cement-treated subbase.

Design of LCF Mixture to Achieve Required Structural Properties

<u>Mixture Design Approach</u>

Recycled crushed concrete was selected as the aggregate for the LCF based on availability, proximity to the construction site, and cost. The RCC aggregate is required to meet P-209 aggregate gradation specifications with a target gradation as mid-line of the P-209 gradation curve. Class C fly ash was added to the aggregate as a filler to produce a durable (low permeability) yet strong (utilizing internal friction among aggregate particles) aggregate matrix. The Class C fly ash was also added as the primary component of pozzolanic strength. Hydrated lime was added as an activator for the fly ash to maximize the pozzolanic reaction. Several trial mixtures were prepared at a design compaction energy of 98 percent of ASTM D 1557. The selected mixture deign consisted of 86.5 percent RCC, 10 percent Class C fly ash, 3 percent hydrated lime, and 0.5 percent Portland cement, which is used to assist in nucleating the pozzolanic and cementitious reactions within the mixture.

The mixture design selected provided a mixture that produced a slow strength gain as shown in Figure 2. The target strength after 6-months of field cure is between 400 and 600 psi. This curing effect was modeled in the laboratory by curing the mixtures at 113 ^{0} F for 45-days. This correlation between laboratory and field curing is based on previous research (Currin et al., 1976) and a literature review (Little, 1998). The target compressive strength after one-year of service is between about 800 and 1,200 psi. According to Figure 2, these targets will be achieved based on the intermediate or average data plot. It is important to remember that a long, steady strength development that will continue well beyond one-year is the goal. Such a process will minimize volume change due to shrinkage and will maximize autogenous healing, which will in turn limit fatigue-cracking damage. Furthermore, the goal for the LCF is to provide acceptable strength and load-carrying ability without becoming too rigid. A very rigid subbase below the PCC slab exacerbates the edge and corner stresses induced by temperature curing and warping of the slab, whereas, a less rigid subbase provides a “cushion” effect reducing such stresses.

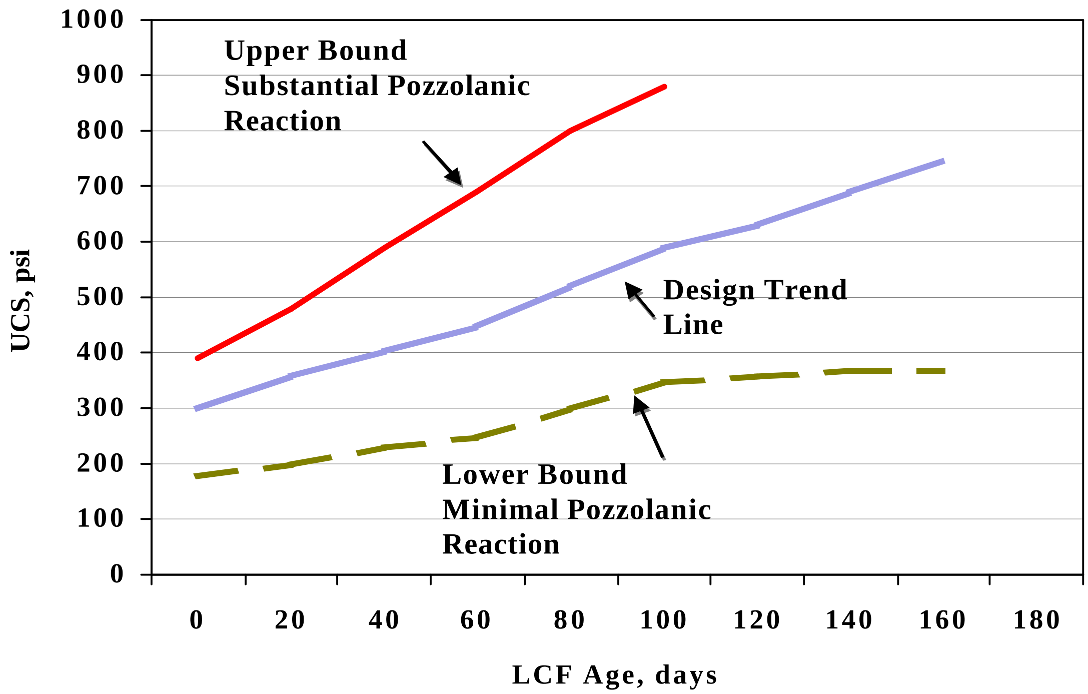

Figure 2. Pozzolanic Strength Bands Developed from Approximately Sixty Laboratory Fabricated Samples Cured at 73 ^oF and 113 ^oF.

Microstructural Analysis of Mixture Design

A petrographic analysis was performed on laboratory samples that were cured for 4-days at 73 ^{o}F in a moisture cure environment and then transferred to a 113 ^{o}F curing room for 45 days. This approximates somewhere between 6-months and one-year of field cure. Samples were compacted to 98 percent ASTM D 1557 compaction energy. The three specimens evaluated were tested for compressive strength prior to petrographic analysis, and had compressive strength of between 464 psi and 967 psi.

The petrographic analysis revealed that the LCF matrix:

- Was dense and should produce a low permeability and therefore durable matrix.
- Contained an air void structure within the pozzolanic matrix that is discontinuous and not conducive to moisture migration which can affect long-term durability.
- Contained a considerable amount of unreacted material (primarily lime and fly ash) indicating the potential for further long-term pozzolanic reaction and continued strength gain at a controlled rate.
- Should maintain continued pozzolanic strength gain which should improve durability with time (by developing a denser matrix) and contribute to autogenous healing.

Field Validation of LCF Mixture Design

The LCF was produced in a central plant pugmill operation. The quantity of each component added was carefully monitored in the plant mixing operation. Approximately four months prior to construction, five mixture designs, the optimum mixture design according to the laboratory testing and four variations thereof, were prepared in the actual field plant under field conditions. Based on the field validation testing, the design mixture was altered slightly by increasing the Portland cement component from 0.5 percent by weight to 1.0 percent by weight. The role of the Portland cement is to initiate the nucleation of the pozzolanic process. The final mixture design following mix plant calibration was then: 86 percent RCC, 10 percent Class C fly ash, 3 percent hydrated lime, and 1.0 percent Portland cement.

In Figure 3, the diamonds represent unconfined compressive strengths determined after 7, 14, and 28 days of cure at 73 ^{0}F and after 100 days of cure at 113 ^{0}F (accelerated cure). Samples were fabricated at the plant. The rate of strength gain roughly coincides with the upper bound.

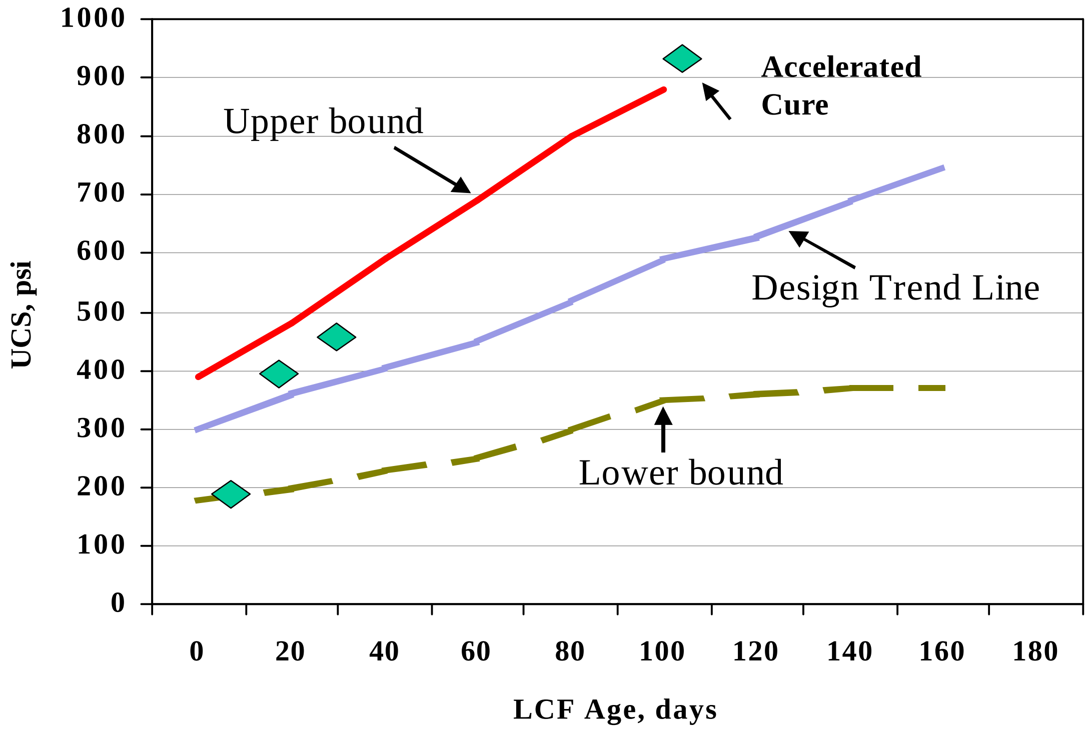

Figure 3. Comparison of Compressive Strengths from LCF Samples Fabricated from Field Batch Plant and Strength Trend Lines.

Nature of Class C Fly Ash

The mixture design and laboratory study determined that strengths could be achieved that meet target values and the field validation substantiated that these strengths could be achieved in the field using the central plant pugmill mixing process. Prior to implementing the approach, however, the design team looked to other projects which used either LCF or lime fly ash (LFA) mixtures. Of course, the BIAH experience with LCF was the primary source, but others included experience from Brown & Brown Contactors of Salina, Kansas. Brown & Brown is one of the largest stabilization and reclamation contractors in the United States. They routinely achieve compressive strengths of between 200 and 600 psi with 2 percent lime and 4 percent fly ash and strengths of between about 600 and 1000 psi with 2 percent lime and 6 to 8 percent fly ash in the blend. The Mississippi Department of Transportation, MDOT, uses LFA mixtures extensively as a structural base. MDOT typically uses between 3 and 4 percent hydrated lime with between 8 and 12 percent fly ash in their LFA bases. A recent review of the Mississippi design (Little, 2001) shows that MDOT typically achieves compressive strengths of between about 800 and about 1,500 psi. Little (2001) evaluated the in situ resilient moduli of the LFA bases using the Falling Weight Deflectometer (FWD) and found the in situ moduli to vary widely but to typically reach a level exceeding 1,000,000 psi.

The design team used the American Coal Ash Association's (ACAA) Pavement Manual as a general guide for designing the pozzolanically stabilized mixture (PSM). The ACAA

Manual was a valuable resource and provided three established correlations between resilient modulus and compressive strengths for fine-grained, sandy, and coarse-grained pozzolanic mixtures.

In order to achieve the desired pozzolonic performance, the design team specified Class C fly ash. Several ashes were evaluated within the market local to BIAH. The W.A. Parish ash was selected. This ash has a total pozzolan content (sum of silica dioxide, aluminum oxide, and ferric oxide) of approximately 58 percent, a CaO content of approximately 26 percent, and a loss on ignition of only 0.08 percent. These quantities easily meet ASTM C 618 requirements for Class C fly ash and are compatible with a strong pozzolanic reaction potential. Furthermore, the Parish ash has a very low sulfur trioxide content (less than 2 percent) eliminating the worry of the growth of expansive calium-aluminate-sulfate-hydrate crystals.

The W. A. Parish (Class C) fly ash was used in the LCF pavement of runway 9-27 and supporting taxiways, SA and SB. Even though fly ash characteristics change with time and coal source, the basic characteristics of the Class C, W. A. Parish ash have remained fairly constant.

Class C fly ash was selected over Class F as experience from the sources considered indicates a more reliable pozzolanic reaction with Class C ash. However, a drawback of Class C ash is the rapid reaction once hydrated. Therefore, as a precaution, the specifications required the compaction of all LCF material to 98 percent of ASTM D 1557 within two hours of production.

Approximation of Design Resilient Modulus from Unconfined Compressive Strengths

A resilient modulus is used in pavement design, not a monotonic or “static” modulus. In the resilient modulus test for base materials (AASHTO T-294-94), the load application attempts to mimic the load dwell time and stress state applied within the pavement layer being tested. The response of a material to dynamic loading, which simulates actual conditions, can be considerably different than the response under monontonic loading or “static” testing conditions. This difference is most apparent in viscoelastic materials such as hot mix asphalt and unbound aggregates, but is also important in pozzolanically stabilized layers.

Because resilient modulus testing is time consuming, it was necessary to make design predictions of moduli based on unconfined compressive strengths of the LCF. Since long-term (one-year and greater) resilient moduli were needed for design, it was necessary to determine the compressive strength of samples subjected to accelerated cure, and then approximate the concomitant design resilient moduli based on correlations. Unfortunately, the range in correlations is extremely wide. Approximately 15 correlations between compressive strength and resilient moduli were found in the literature and evaluated. The most promising of these correlations are listed in Table 3.

Table 3. Summary of Models Correlating Unconfined Compressive Strength to Modulus.

Model	**Unconfined Compressive Strength, psi**	**Predicted Modulus, psi**	**Source of Model and Comments**
E_{psm} (ksi) = 500 + UCS (psi) Where UCS = unconfined compressive strength	100 500 1,000	600,000 1,000,000 1,500,000	American Coal Ash Pavement Manual, (1990)
$E = 2240\ UCS^{0.88} + 1100$ (all in MPa)	100 500 1,000	391,000 1,121,000 1,930,000	Australian Road Research Laboratory (1998)
$E = 33 \times w^{1.5}\ UCS^{0.50}$ (all in psi) Where w = unit weight in pcf	100 500 1,000	500,046 1,119,076 1,582,613	Model for reinforced concrete, Barenberg, (1977)
E = 1,200 UCS	100 500 1,000	120,000 600,000 1,200,000	Model for coarse-grained sandy material, Barenberg, (1977)
$E = 440\ UCS + 0.28\ (UCS)^2$ (all in psi)	100 500 1,000	46,800 290,000 720,000	Model for cement stabilized fine-grained soils, Barenberg, (1977)
TTI	100 500 1,000	60,000 125,000 180,000	TTI investigation of cement treated bases (unpublished)
E (ksi) = 0.124 UCS (psi) + 9.98	100 500 1,000	22,400 72,000 134,000	Thompson model for lime stabilized soils (1986)
$E = 0.25\ (UCS)^2$ (all in psi)	100 500 1,000	2,500 60,000 250,000	McClelland Engineers for LCF Mixtures on Runway 9-27 (unpublished)

A dynamic modulus test was performed on four LCF cores. The magnitude and wave pulse of the dynamic stress in this experiment simulates the stress imparted by a moving aircraft wheel in the LCF layer, based on layered elastic analysis of the pavement structure. The four cores fell into two groups. The first group represented compressive strengths in the 300 to 350 psi range while the second represented compressive strengths in the 650 to 700 psi range. The concomitant dynamic moduli of the 300 to 350 psi compressive strength samples were 500,000 to 610,000 psi. The concomitant dynamic moduli for the 650 to 700 psi compressive strength were 950,000 to 1,100,000 psi. These correlations compare favorably with the Barenberg (1977) model, which is expressed in row five of Table 3. Resilient modulus testing according to ASTM-T-294-94 was performed on Portland cement treated recycled crushed concrete (CTRCC) over a range of stress states. The CTRCC has the same gradation of crushed concrete aggregate as the LCF. Correlations between modulus and compressive strength for the CTRCC (a smaller material to LCF) once again verified that the Barenberg (1977) model is appropriate.

<u>Resilient Modulus versus Monotonic Modulus</u>

One difficulty with employing the compressive strength – modulus correlations presented in the literature, some of which are summarized in Table 3, is to determine the nature of the modulus measured. A common measure of modulus in such correlations is the secant modulus under a monotonic load where stress is increased monotonically until the failure strength is exceeded. Secant moduli from monotonic loading can vary significantly from resilient moduli measurements such as those made following either the dynamic modulus protocol or AASHTO T 294-94, where the load pulse is applied rapidly and in a manner to mimic a moving wheel load. Based on a careful analysis of the correlations listed in Table 3, it is apparent that the Barenberg (1977) model for stabilized, coarse-grained sandy material offers both a correlation that is roughly near the middle of the other range of correlations. Furthermore, it agrees reasonably well with laboratory testing. For this reason, the design resilient modulus was estimated from the unconfined compressive strength as E_r = 1,200 (UCS). Where UCS is the unconfined compressive strength in psi.

In order to illustrate the need to measure resilient modulus rather than monotonic modulus, consider a silty subgrade whose resilient modulus determined to be approximately 10,000 psi at a deviatoric stress of 6 psi is measured according to AASHTO T 294. The unconfined compressive strength of this soil is approximately 10 psi. It is obvious that some of the power correlations between compressive strength and modulus listed in Table 3 substantially under predict modulus of low strength materials. While these correlations are probably only valid over select ranges, the Barenberg (1977) correlations cited, seems to be reasonable between compressive strengths of about 100 psi and about 1,500 psi. The correlation established by the American Coal Ash Association (ACAA) for pozzolanically stabilized mixtures is considerably more non-conservative in its prediction of modulus than the selected model.

Quality Control of LCF Mixtures

Mixture quality was monitored through a strict quality assurance program that consisted of gradation control of the recycled crushed concrete, validation of the mixture design through trial mixes in the pugmill, and continued monitoring of strength during the production of the LCF. Strength monitoring consisted of compressive strength testing at 7, 14, and 28-day of 73 ^{o}F cure as well as 28-days of 113 ^{o}F cure. Samples were fabricated to achieve 98 percent of ASTM D 1557 compaction. Samples were wrapped in plastic and then placed in a plastic bag to insure retention of moisture during curing. Minimum compressive strengths were established as 250 psi, 275 psi, 350 psi, and 400 psi for the 7, 14, and 28-day normal cure and 28-day accelerated cure, respectively. Representative material was collected from every 500 tons of LCF mix produced for pavement construction to fabricate compressive strength specimens. Moisture content, mix proportions, and density were also checked for every 500 ton quantities.

Quality control specifications consisted of thickness placement control, compaction to a minimum density of 98 percent of ASTM D 157, and grade control. The contractor was required to follow strict requirements for placing and compacting the mixture to assure that the LCF was placed and compacted within two hours of production. This is critically important as the Class C ash will set rapidly, and if compaction is not achieved within the two-hour window, the ultimate strength will be compromised.

Compaction of each lift to the 98 percent ASTM D 1557 density was easily achieved.

Validation of Mixture Properties

Accelerated Cure Strengths

As discussed under the section on mixture design, accelerated curing at 113 ^{o}F for 28 days mimics approximately 100 days of field curing at approximately 73 ^{o}F while curing for approximately 40 to 45 days at 113 ^{o}F roughly corresponds to approximately six months in the field at 73 ^{o}F (Currin et al., 1976, Little, 1998). Data from field mixed but laboratory compacted and accelerated cure samples are plotted in Figure 4. Accelerated cure laboratory data represent curing at 113 ^{o}F for 28-days and 113 ^{o}F for 42-days. The 28-day laboratory cure data (610 psi, average of 45 data points) are plotted at the 100 day position. The 42-day data (785 psi, average of 35 data points) are plotted at the 180-day position. These data are consistent with the expected or design trend line in Figure 4.

Field Cores

Twenty-five field cores were extracted from random positions in the LCF pavement. The cores, though randomly located, were uniformly spread across the length and breadth of the pavement. The average age of the first nine cores was 19-days at the time of test and the average strength of these nine cores was 365 psi. A second set of four cores with an age of between 40 and 42-days had an average strength of 530 psi. Finally, a third set of 13 cores with an average age of 60-days (ranging in age from 56 to 62-days) had an average strength of 620 psi. These points are also plotted in Figure 3.

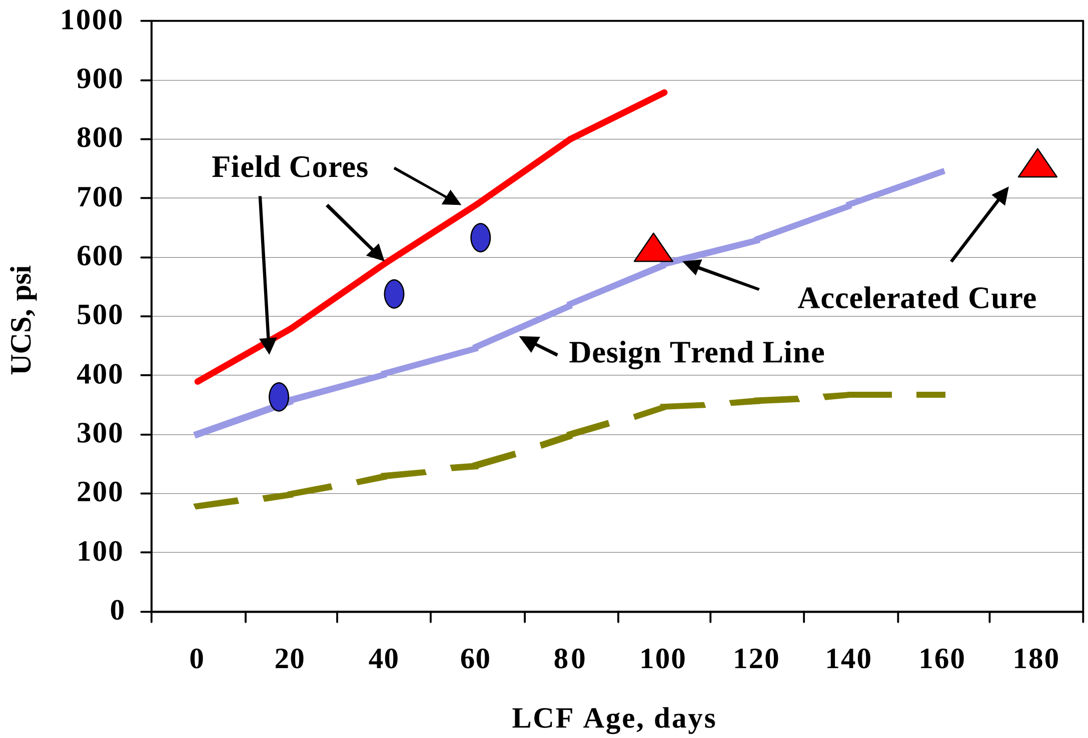

Figure 4. Comparisons of Accelerated Cure LCF Mixtures and Field Cores with Respect to Trend Lines.

The clear trend from the laboratory fabricated, accelerated cure samples and the field cores is that the LCF mixtures are gaining strength in the field at or slightly above the expected trend line, the middle line in Figure 3. The rate of strength gain helps ensure a controlled and steady strength gain rate and the potential of meeting the one-year target of between 1,000 psi and 1,500 psi.

Conclusions

A mixture of lime, cement, and fly ash (LCF) was used as a subbase on taxiway WP to support a Portland cement concrete pavement surface. The LCF layer was engineered to provide a target strength at the end of one year of service of about 1,000 psi and a concomitant resilient modulus of about 1,000,000 psi. The LCF was designed to gain strength in a slow, controlled manner in order to reduce shrinkage cracking and to optimize autogenous healing over the life of the pavement. Since the LCF is a subbase to support a PCC surface, an overly rigid subbase was considered undesirable in the design phase.

The mixture design approach used achieved the desired results based on field core test data. The LCF mixture gained strength in accordance with the trend lines that predict strength gain based on laboratory testing. The LCF uses recycled crushed concrete as the aggregate source and locally available, Class C fly ash. This is the first use of LCF as a subbase for a PCC pavement.

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